11-064904

IH CONTENTS B08-31-99

Edit listed contents to suit job. Eliminate note for extra copies for jobs with no piling.

Include this cover sheet with submittal to District at PS&E, and also include the submittal to HQOE at Expedite. Update as necessary.

INFORMATION HANDOUT
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PROJECT:
CONTRACT NO.: 11-064901

SUBMITTAL TYPE:	re. PS&E (to District)	EXPEDITE (to HQOE with copy to District)	POST EXPEDITE (to HQOE with copy to District)
SUBMITTAL DATE:	9/11	9/11	

STRUCTURES SPEC ENGINEER: Denise Blakesley
PHONE: CALNET 498-8577 or (916) 227-8577

Contents	Bridge Number(s)
Foundation Recommendations , 3/18/02, 6/19/02	57-0884F
Foundation Reviews , 5/24/02, 8/28/02	57-0884F
~~Materials Reports~~	
~~Pile Indicator Test Reports~~	
~~Driveability Analysis~~	
~~As Built Log of Test Borings (not in the contract plans)~~	
As Built Pile Driving Records	57-0884F
Revised Pile Data Table 8/14/02	57-0884F
Northeast connector Widening: Amendment 8/26	

FOUNDATION REVIEW

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

Date: 8/28/02

: **Structure Design**
1. Design
2. R.E. Pending File
3. ~~Specifications & Estimates~~
4. File

Geotechnical Services
1. GD - North ; South ; West
2. GS File Room

Northeast Gm. OC.
Structure Name

11 - SD - 15/8 - 10.83
District County Route km Post

District Project Development
District Project Engineer

11-064901 57-0884F
E.A. Number Structure Number

oundation Report By: Z. Yazdani
Dated: 8/14/02 ; 6/19/02 ; 3/18/02

eviewed By: L. Wang (SD) R. Price (GS)

eneral Plan Dated: 8/21/02 Foundation Plan Dated: 8/20/02

☒ No changes. ☐ The following changes are necessary.

Value of bearing capacity, as shown on Foundation Recommendations (dated 3/18/02), should read, ... allowable bearing capacity of 202 kPa Ultimate is 606 kPa

FOUNDATION CHECKLIST

✓ Pile Types and Design Loads	✓ Footing Elevations, Design Loads, and Locations	✓ Effect of Fills on Abutments and Bents
✓ Pile Lengths	✓ Seismic Data	✓ Fill Surcharge
✓ Predrilling	✓ Location of Adjacent Structures and Utilities	✓ Approach Paving Slabs
✓ Pile Load Test	✓ Stability of Cuts or Fills	✓ Scour
✓ Substitution of H Piles For Concrete Piles ☐ Yes ☒ No	✓ Fill Time Delay	✓ Ground Water
		✓ Tremie Seals/Type D Excavation

Structure Design ____ Bridge Design Branch No. 18 ____ Geotechnical Services

ev. 06/02

State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. ELIAS KURANI
Senior Bridge Engineer
Office of Structural Design - South
Design Branch 18/MS#9

Attention: Mr. Linan Wang

Date: August 26, 2002

File: 11-SD-15&8
R10.3/R12.1
EA 11 - 064901
Northeast Connector
Bridge No. 57-0884F

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South

Subject: Northeast Connector Widening: Amendment.

A foundation report for the subject structure was issued on June 19, 2002. A revised pile data table for Bents 2 and 3 based on revised loadings was transmitted to you on August 14, 2002. This letter presents specific elevations with regard to the drilling of the pile shafts for Bents 2 and 3.

In our referenced June 19, 2002 report, we have stated that "drilling ahead of the casing, especially within the upper loose and medium dense zone should be avoided to reduce caving and creating of voids". Approximate elevations that pertain to this statement are as follow:

Bent 2 - from elevation 21.03 to elevation 9.35
Bent 3 - from elevation 17.38 to elevation 9.05

Additionally, in our foundation report, we have stated that "drilling ahead of casing will likely be required in order to advance casing within the gravelly and cobbly zone and within the bedrock". Approximate elevations that pertain to this statement are as follows:

Bent 2 - from elevation 9.35 to elevation 8.05
Bent 3 - from elevation 9.05 to elevation 6.95

If you have any questions or comments regarding this report, please call Moussa H Jandal at (858) 467-4061 (Calnet 734-4061) or Zia Yazdani at (858) 467- 4054 (Calnet 734-4054).

Prepared by Moussa H Jandal on August 26, 2002

Moussa H Jandal
Moussa H Jandal
Transportation Engineer (Civil)

Zia Yazdani
Zia Yazdani
Associate Materials and Research Engineer

REGISTERED PROFESSIONAL ENGINEER
ZIAULLAH YAZDANI
No. 2119
Exp. 3-31-0?
GEOTECHNICAL
STATE OF CALIFORNIA

cc:
John Ehsan
Joe Egan
RGES.02

State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. ELIAS KURANI
Senior Bridge Engineer
Office of Structural Design - South
Design Branch 18/MS#9

Attention: Mr. Linan Wang

Date: August 14, 2002

File: 11-SD-15&8
R10.3/R12.1
EA 11 - 064901
Northeast Connector
Bridge No. 57-0884F

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South

Subject: Northeast Connector Widening: Revised Pile Data Table.

In accordance with your request, we have developed the revised pile shaft length for Bent 2 as a result of the revised loading that you transmitted to us recently. The revised loading shows a load increase of 25 percent in compression and a decrease of about 6 percent in tension. Based on these revised loads we have calculated an additional shaft length of about 6.1m (20 feet) at Bent 2. The pile data for Bent 3 remain unchanged. Table 3 of our June 19, 2002 report has been revised to reflect the referenced changes and is shown on the following page.

Table 3: Revised Pile Data Table, Bridge No. 57-0884F, Bents 2 and 3.

BENT LOCATION Type and Diamete)	NOMINAL RESISTANCE		INTENDED LENGTH OF ROCK SOCKET m (ft)	PERMANENT CASING SPECIFIED TIP ELEVATION m (ft)	DESIGN PILE TIP ELEVATION m (ft)	SPECIFIED PILE TIP ELEVATION m (ft)
	COMPRESSION kN (tons)	TENSION kN (tons)				
Bent 2 CIDH 1.8 m (6 ft)	12,500 (1,405)	4,500 (506)	27.44 (90)	9.35 (30.7)	-18.09[1,2] (-59.3)	-18.09[1,2] (-59.3)
Bent 3 CIDH 1.8 m (6 ft)	10,000 (1124)	0	21.34 (70)	9.05 (29.7)	-12.29[1] (-40.3)	-12.29[1] (-40.3)

Notes: Design tip elevation is controlled by the following demands: 1) Compression; 2) Tension; 3) Lateral Loads

If you have any questions or comments regarding this report, please call Moussa H Jandal at (858) 467-4061 (Calnet 734-4061) or Zia Yazdani at (858) 467- 4054 (Calnet 734-4054).

Prepared by Moussa H Jandal on August 13, 2002

Moussa H. Jandal

Moussa H Jandal
Transportation Engineer (Civil)

Zia Yazdani

Zia Yazdani
Associate Materials and Research Engineer

REGISTERED PROFESSIONAL ENGINEER
ZIAULLAH YAZDANI
No. 2119
Exp. 3-31-05
GEOTECHNICAL
STATE OF CALIFORNIA

cc:

John Ehsan
Joe Egan
RGES.02

Memorandum

To: MR. ELIAS KURANI
Senior Bridge Engineer
Office of Structural Design - South
Design Branch 18/MS#9

Attention: Mr. Linan Wang

Date: June 19, 2002

File: 11-SD-15&8
R10.3/R12.1
EA 11 - 064901
Northeast Connector
Bridge No. 57-0884F

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South

Subject: Northeast Connector Widening: Foundation Recommendations.

INTRODUCTION

Following your request, the Office of Geotechnical Design – South conducted a foundation investigation for the proposed widening of the subject connector from Abutment 1 to Bents 2 and 3. Our investigation consisted of a review of the existing as-built plans and geologic maps, site reconnaissance, limited geologic surface mapping, subsurface investigation, engineering analysis, and the writing of this report. For this report, in addition to our Borings B2-B2 and B3-B3, we utilized archived logs of test borings that were drilled during the exploration programs of 1973 and 1979. Along with your request, we have received from you the proposed structure layout sheets (in scales of 1:1000 and 1:500) and cross sections that were used in our fieldwork, engineering analyses, and the preparation of this report.

PROJECT LOCATION

The project site is located in the City of San Diego, California. It generally involves the Northeast Connector from southbound I-15 to eastbound I-8. It comprises the section of the bridge that stretches from the northern abutment to about Bent 3 (Station 121+40 to about Station 122+40 "SE" Line).

GEOLOGY

The project site lies within the Peninsular Ranges Geomorphic Province of California. The area where the existing Bridge No. 57-0884F is located lies generally at the confluence of Murphy Creek and the San Diego River channel. Therefore, the project site is underlain by the unconsolidated Holocene and older Quaternary granular alluvial and slopewash deposits. The basement (bedrock) is composed of sandstone with layers or/and lenses of claystone and siltstone (Kennedy and Peterson, 1975).

SEISMICITY

The project site is located at about the conjunction of the two geomorphologic units: the southern end of the north south trending Murphy Canyon and east west trending Mission Valley. No known active (Holocene) fault exists within the project area. However, both the aforementioned geomorphologic units are believed to be fault related (Kennedy and Peterson, 1975). The nearest known active fault is the Newport Inglewood -- Rose Canyon Fault Zone believed to be capable of producing an earthquake with a Maximum Credible Magnitude of 7.0 on the Richter scale. It is located about 8.5 km west from the project site. The Newport Inglewood -- Rose Canyon Fault is believed to be capable of generating a Peak Ground Acceleration of about 0.45 g at the project site (Mulchin, 1996). We understand that the ARS curve for the site has been provided to you by the Office of Geotechnical Earthquake Engineering.

LIQUEFACTION

The potential for soil liquefaction during a seismic event on the Newport Inglewood -- Rose Canyon Fault Zone is the greatest seismic-related threat to the project. Our evaluation of subsurface conditions at the location of Bents 2 and 3 (Bridge No. 57-084F) indicates a moderate to high potential for soil liquefaction. This is because the Holocene and older Quaternary alluvial soils at the project site are predominantly composed of loose to medium dense sand and silty sands. In addition, the ground water table at the project site is relatively shallow. It is our understanding that in their memorandum dated May 9, 2002, the Office of Geotechnical Earthquake Engineering has provided you with p-y curves for liquefable soils to be used in the lateral analysis. Additionally, the downdrag force on each pile was estimated by the Office of Earthquake Engineering to be approximately 432 kN (48.5 Tons). This value of the downdrag force was utilized in the development of the required shaft length to be socketed into the sandstone layer.

GROUNDWATER

Groundwater was encountered during this investigation. In order to determine the elevation of the groundwater table and its fluctuation over time, a piezometer was installed in Boring B3-B3. The piezometer was monitored periodically after its installation. In addition, it was field-surveyed and mapped with reference to the layouts stations provided by your office. The measured groundwater elevations and the recording dates are shown in Table 1 below.

Table 1: Groundwater Elevation

Boring No.	Groundwater Elevation	Date Measured
B3-B3	15.10 m	1/10/02
B3-B3	15.09 m	1/14/02

Based on collected data and data from the 1973 and 1979 exploration programs, we determined that an unconfined groundwater condition exists under the project site. The water table is generally at an elevation of about 15.08 m. Fluctuations in the level of groundwater may occur due to factors such as subsurface stratification and rainfall. The level of groundwater may rise by the end of the rainy season.

SUBSURFACE SOIL CONDITIONS

Our surface mapping and subsurface investigation (Borings B2-B2, B3-B3), along with the archived borings of the exploration programs of 1973 and 1979, revealed that Bents 1 and 2 are underlain by alluvium that in turn is underlain by competent bedrock. Based on Boring Logs B2-2 and 1973-B-12, the interface between alluvium and bedrock at the location of Bent 2 was found to be at about an elevation of 9.45 m. Based on Boring B3-B3 the interface between alluvium and bedrock at the location of Bent 3 was found to be at about an elevation of 9.10 m. Alluvium soils consist of sand with scattered gravel, sandy silt and silty sand. Silty deposits generally occur above the elevation of the groundwater table (about 15.08 m), whereas sands were deposited below the groundwater elevation. Based on SPT blow counts, the relative density of alluvial soils was found to range from loose to medium dense. At about the contact between the alluvium and bedrock, an alluvial gravelly and cobbly zone of variable thickness was found. Thus, the SPT blow counts within that zone should be considered affected by gravel. In Boring B2-B2 the gravelly/cobbly zone (layer) was logged to be about 0.7 m thick and in Boring B3-B3 the gravelly zone was logged to be about 1.5 m thick. Bedrock consists of weathered sandstone with claystone and siltstone interbeds. From the geotechnical engineering standpoint, based on core samples index tests and SPT blow counts, we found the bedrock to be competent. In addition, we found that claystone layers or interbeds could not be penetrated by the SPT sampler. For groundwater conditions at the location of Bents 2 and 3, reference is directed to the section titled "Groundwater" on page 2 of this report.

FOUNDATION RECOMMENDATIONS

Plumb, 1.8 m (6 ft) diameter, Cast-in-Drilled-Hole (CIDH) Piles can be used to support additions to Bents 2 and 3 of Bridge No. 57-0884F. CIDH Pile capacities were calculated utilizing procedures detailed in the Federal Highway Administration's Load Transfer for Drilled Shafts in Intermediate Geomaterials publication (FHWA, 1996). In our analysis we utilized the procedure developed by Mayne and Harris for a Type 3 geomaterial (very dense granular geomaterial with SPT blow count of 50 – 100 blows/0.3 m). The sandstone (Type 3 Geomaterial) was assigned a corrected N-value of 65 to a depth of 10 m below the top of the stratum, N-value of 75 below 10 m, and a base layer N-value of 100. Based on our analysis, CIDH shafts lengths are presented in Table 3, Pile Data Table shown below.

It is recommended that permanent steel casing be placed into bedrock to facilitate construction of the drilled shafts, to prevent caving of loose soils and gravel/cobble layers into the pile borings and to seal off the entry of ground water into the pile borings. Permanent steel casing should be driven to at least the top of the gravel/cobble layer using a vibratory hammer. Drilling slightly ahead of casing in the basal gravel/cobble zone and within bedrock will most likely be required. Our analysis assumes no additional axial geotechnical capability for permanent steel casing that will be installed to aid in the construction of the CIDH piles. The practice of drilling ahead of the casing in the alluvial soil layer, before dropping the casing into place, is not recommended. This is because any caving of loose soils would create voids between the casing and surrounding soil, which could impact the lateral capacity of the piles.

Table 3: Pile Data Table, Bridge No. 57-0884F, Bents 2 and 3.

BENT LOCATION Type and Diamete)	NOMINAL RESISTANCE		INTENDED LENGTH OF ROCK SOCKET m (ft)	PERMANENT CASING SPECIFIED TIP ELEVATION m (ft)	DESIGN PILE TIP ELEVATION m (ft)	SPECIFIED PILE TIP ELEVATION m (ft)
	COMPRESSION kN (tons)	TENSION kN (tons)				
Bent 2 CIDH 1.8 m (6 ft)	9650 (1085)	4,800 (540)	21.34 (70)	9.35 (30.7)	-11.99[1,2] (-39.3)	-11.99[1,2] (-39.3)
Bent 3 CIDH 1.8 m (6 ft)	10,000 (1124)	0	21.34 (70)	9.05 (29.7)	-12.29[1] (-40.3)	-12.29[1] (-40.3)

Notes: Design tip elevation is controlled by the following demands: 1) Compression; 2) Tension; 3) Lateral Loads

Axial compression values noted in the tables above are based on skin friction only within the bedrock. End bearing was not considered due to working below the water table and the possibility that cleaning out the bottom of pile borings effectively may be rather difficult at depth and may make it difficult to realize substantial end bearing using Caltrans standard pile vertical deflection criteria of 13 mm (0.5 inch). In addition, if the pile tip elevation is controlled by lateral demands, the designer is responsible to present correct foundation data, governed by lateral control, on the foundation plans.

The Pile Data Table above includes intended length of the rock socket at each bent location. The intended length of the rock socket should be measured from the bottom of the permanent casing down to the pile specified tip elevation. The permanent casing should be seated into rock approximately 0.61 m (2 ft).

CONSTRUCTION CONSIDERATIONS

As mentioned above, we recommend installation of permanent casing to be placed into bedrock to prevent caving of loose soils and gravelly/cobbly material into pile borings and help seal off groundwater from entering the excavations once seated into the bedrock. The use of a vibratory hammer to place steel casing down to a level close to casing specified tip elevation would facilitate pile construction and effectively reduce creation of voids along the pile length by undesirable caving of loose soils and gravelly/cobbly material. Drilling ahead of the casing, especially within the upper loose and medium dense zone should be avoided to reduce caving and creation of voids, thus compromising lateral pile capacity. We anticipate center relief drilling to facilitate casing advancement. Hard slow drilling through gravelly and cobbly zones and very hard claystone is anticipated during installation of permanent casing and CIDH piles (rock sockets). Drilling ahead of casing will likely be required in order to advance casing within the gravelly and cobbly zone and within bedrock. Once casing is seated into bedrock, drilling for the rock sockets can be completed.

Groundwater should be anticipated at relatively shallow depth. Static groundwater was determined to be at an elevation of about 15.08 m. The wet method is advised for CIDH pile construction. The bottom of both excavations should be cleaned of loose debris before placing concrete.

CORROSIVITY

In order to determine the corrosivity of subsurface soils at the location of Bents 2 and 3, we obtained a soil sample that was sent to the Caltrans Laboratory for analysis. The test results indicated a ph of 7.3 and minimum resistivity of 3050 ohm-cm. Based on the corrosion test result, and our experience in the area, we conclude that soils at the locations of Bents 2 and 3 are non-corrosive.

If you have any questions or comments regarding this report, please call Jeff Tesar at (858) 467-2716 (Calnet 734-2716) or Zia Yazdani at (858) 467- 4054 (Calnet 734-4054).

Prepared by Jeff Tesar, June 20, 2002

J. Tesar.

Jeff Tesar
Associate Engineering Geologist

Zia Yazdani

Zia Yazdani
Associate Materials and Research Engineer

Geotechnical Branch 11

REGISTERED GEOLOGIST
JEFF J. TESAR
No. 2137
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
ZIAULLAH YAZDANI
No. 2119
Exp. 3-31-05
GEOTECHNICAL
STATE OF CALIFORNIA

ATTACHMENTS

1. Logs of Borings

REFERENCES

1. Kennedy and Peterson, Geology of the San Diego Metropolitan Area, California, La Mesa Quadrangle, Bulletin 200, 1975.

2. Mualchin, California Seismic Hazard Detail Index Map 1996.

3. O'Neill, Townsend, Hassan, Buller, Chan, Load Transfer for Drilled Shafts in Intermediate Geomaterials, Publication No. FHWA-RD-95-172, November 1996.

JT

cc:
John Ehsan
Joe Egan
RGES.02

FOUNDATION REVIEW

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

Date: 5/24/02

To: **Structure Design**
1. Design
2. ~~R.E. Pending File~~
3. Specifications & Estimates
4. ~~File~~

Northeast Corr. O.C.
Structure Name

11 - SD - 15/08 - 10.85
District County Route km Post

Geotechnical Services
1. GD - North
2. GD - South
3. GD - West

District Project Development
District Project Engineer

11-06491 57-0884F
E.A. Number Structure Number

Foundation Report By: J. Teran
Dated: 3/18/02

Reviewed By: L. Wang (SD) R. Price (GS)

General Plan Dated: 5/9/02
Foundation Plan Dated: 5/9/02

☐ No changes. ☒ The following changes are necessary.

(1) Standard Specification 49-1.05 shall apply "if" necessary. However, jetting is not allowed. If drilling is needed, it should be limited to 3m above specified tip elev.

(2) Standard Specification 49-1.05 shall apply.

FOUNDATION CHECKLIST

✓ Pile Types and Design Loads	✓ Footing Elevations, Design Loads, and Locations	✓ Effect of Fills on Abutments and Bents
✓ Pile Lengths	✓ Seismic Data	✓ Fill Surcharge
✓ Predrilling	✓ Location of Adjacent Structures and Utilities	✓ Approach Paving Slabs
✓ Pile Load Test	✓ Stability of Cuts or Fills	✓ Scour
✓ Substitution of H Piles For Concrete Piles ☐ Yes ☒ No	✓ Fill Time Delay	✓ Ground Water
		✓ Tremie Seals/Type D Excavation

Structure Design — Section 12
Bridge Design Branch No.

Geotechnical Services

Rev. 06/01

State of California Business, Transportation and Housing Agency

Memorandum

To: MR. ELIAS KURANI
Senior Bridge Engineer
Office of Structural Design - South
Design Branch 18/MS#9

Attention: Mr. Linan Wang

Date: March 18, 2002

File: 11-SD-15/Friars Rd.
R10.3/R12.1
EA 11 - 064901
Northeast Connector
Bridge No. 57-0884F

Caltrans Metric

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South

Subject: Foundation Recommendations.

INTRODUCTION

These Foundation Recommendations serve as a revised report to our Geotechnical Investigation Report titled "Interstate 15 Northeast Connector: Proposed Widening of Abutment 1 and Construction of Retaining Wall RW1" dated February 19, 2002.

Following your request, the Office of Geotechnical Design – South conducted a foundation investigation for the proposed widening of Abutment 1 and construction of Retaining Wall RW1. The construction of Retaining Wall RW1 is associated with the widening of the aforementioned abutment. Our investigation consisted of a site reconnaissance, review of the existing as-built plans and geologic maps, limited geologic mapping, subsurface investigation, engineering analysis, and the writing of this report. We were not able to drill at the location of planned addition to the Abutment 1. Therefore, for this report, in addition to our Borings RW1-B1 and RW1-B2, we utilized archived logs of test borings that were drilled during the exploration programs of 1973 and 1979. Along with your request, we have received from you the proposed structure layout sheets (in scales of 1:1000 and 1:500) and cross sections that were used in our fieldwork, engineering analyses, and the preparation of this report.

PROJECT LOCATION

The existing Abutment 1 of Bridge No. 57-0884F is located at the northern limit of the Northeast Connector from southbound Interstate 15 (I-15) to eastbound Interstate 8 (I-8). The project site is located in the City of San Diego, California. It generally involves the Northeast Connector from southbound I-15 to eastbound I-8, from approximately Station 120+20 to about Station 121+40 "SE" Line.

GEOLOGY

The project site lies within the Peninsular Ranges Geomorphic Province of California. The area where the existing Abutment 1 is located and Wall RW1 is planned to be located lies generally at the confluence of Murphy Creek and the San Diego River channel. Therefore, with the exception of the localized top fill mantle, the project site is underlain by the unconsolidated Holocene and older Quaternary granular alluvial and slopewash deposits. The basement (bedrock) is composed of sandstone with layers or/and lenses of claystone and siltstone (Kennedy and Peterson, 1975).

SEISMICITY

The project site is located at about the conjunction of the two geomorphologic units: the southern end of the north south trending Murphy Canyon and east west trending Mission Valley. No known active (Holocene) fault exists within the project area. However, both the aforementioned geomorphologic units are believed to be fault related (Kennedy and Peterson, 1975). The nearest known active fault is the Newport Inglewood -- Rose Canyon Fault Zone believed to be capable of producing an earthquake with a Maximum Credible Magnitude of 7.0 on the Richter scale. It is located about 8.5 km west from the project site. The Newport Inglewood -- Rose Canyon Fault is believed to be capable of generating a Peak Ground Acceleration of about 0.45 g at the project site (Mulchin, 1996). We understand that the ARS curve for the site has been provided to you by the Office of Geotechnical Earthquake Engineering.

LIQUEFACTION

The potential for soil liquefaction during a seismic event on the Newport Inglewood -- Rose Canyon Fault Zone is the greatest seismic-related threat to the project. Liquefaction could initiate settlement of the fill approach ramp that was placed on the underlying alluvial soils that are moderately to highly susceptible to liquefaction. Liquefaction could also impact the structural integrity of the existing and proposed structures. Our evaluation of subsurface conditions at the location of the Abutment 1 and the alignment of the proposed Wall RW1 indicates a moderate to high potential for soil liquefaction. This is because the Holocene and older Quaternary alluvial soils at the project site are predominantly composed of loose to medium dense sand and silty sands. In addition, the ground water table at the project site is relatively shallow.

GROUNDWATER

Groundwater was encountered during this investigation. In order to determine the elevation of the groundwater table and its fluctuation over time, piezometer was installed in Boring RW1-B1. The piezometer was monitored periodically after its installation. In addition, the piezometers was field-surveyed and mapped with reference to the layouts stations provided by your office. The measured groundwater elevations and the recording dates are shown in Table 1 below.

Table 1: Groundwater Elevation

Boring No.	Groundwater Elevation	Date Measured
RW1-B1	15.13 m	1/10/02
RW1-B1	15.10 m	1/14/02

Based on collected data and data from the 1973 and 1979 exploration programs, we determined that an unconfined groundwater condition exists under the project site. The water table is generally at an elevation of about 15. 08 m. Fluctuations in the level of groundwater may occur due to factors such as subsurface stratification and rainfall. The level of groundwater may rise by the end of the rainy season.

SITE CONSIDERATIONS, SUBSURFACE SOIL CONDITIONS, AND FOUNDATION CONCLUSIONS AND RECOMMENDATIONS

The foundation of the proposed Retaining Wall RW1 will be embedded in structural backfill material. The pile foundation of Abutment 1 will penetrate through the native alluvium that is comprised of sand and silty sand with scattered gravel and be founded in competent bedrock that is comprised predominantly of sandstone with interbedded claystone. For these soils, based on our subsurface investigation and the explorations conducted in 1973 and 1979, the site geology and our experience in the area, we have established generalized geotechnical parameters that were used in our foundation analyses. These parameters are listed in Table 2 on the following page. The spread footing foundation analyses were based on procedures outlined in the FHWA manual (Cheney and Chassie).

Table 2: Geotechnical Units Design Parameters

GEOTECHNICAL UNIT	COHESION (KPa)	ANGLE OF INTERNAL FRICTION (degree)	MAXIMUM DRY DENSITY (KN/m^3)
Structural Backfill	9.6	36	19.0
Alluvium	9.6	32	19.0
Bedrock	30.0	38	20.4

RETAINING WALL RW1

SITE CONSIDERATIONS

Wall RW1 will be 111.9 m long and 5.5 m in maximum height at the southern limit of the wall, at the location of the Abutment 1, Station 121+31.9. From that station to the north, the wall height will sharply decrease reaching ground level at its northern limit. Based on the Wall RW1 profile, out of the total wall length of 111.9 m, about a 37 m long northern section of the wall will be 3.0 m high or less, and about a 40.0 m long section will be 1.5 m high or less. From about Station 120+20 to 121+31.8, it will parallel to the south the eastern shoulder of the approach ramp of the Northeast Connector from southbound I-15 to eastbound I-8. This interval represents an existing approach ramp to the Bridge No. 57-0884F and was built out of fill materials. The maximum height of the ramp is about 5.0 m. Along this interval, the east-facing ramp embankment slope descends at a general inclination of 1:1.5 vertical to horizontal (V:H). During our geologic mapping, we found the upper 0.3 m thick layer of the slope to be weathered. Based on the layout and cross sections provided by your office, the subject approach ramp is to be widened to accommodate an additional lane. The widening will involve placing of fill on the existing ramp embankment slope and retaining it with the proposed Wall RW1.

SUBSURFACE SOIL CONDITIONS

Our surface mapping and subsurface investigation (Borings RW1-B1, RW1-B2), along with the archived borings of the exploration programs of 1973 and 1979, revealed that the project site, along the alignment of the proposed Wall RW1 and at the location of Abutment 1, is underlain by a fill wedge that in turn is underlain by alluvium. Alluvium deposits are underlain by competent bedrock. The fill wedge is associated with the construction of the Northeast Connector (Bridge No. 57-0884F) and grading of its approach ramp. In Boring RW1-B1 located near the southern limit of planned Wall RW1, the fill and alluvium interface was logged at an elevation of about 20.3 m. At the southern limit of planned Wall RW1 and the location of the existing Abutment 1, the interface, based on Boring RW1-B2, was extrapolated to be at an approximate elevation of 21.0 m. Fill materials consist of silty sand with gravel and cobbles. Based on SPT blow counts, the relative density of the fill was found to be medium dense. Alluvium soils consist of sand with scattered gravel, sandy silt and silty sand. Silty deposits generally occur above the elevation of the groundwater table (about 15.08 m), whereas sands were deposited below the groundwater elevation. Based on SPT blow counts, the relative density of alluvial soils was found to range from loose to medium dense. The interface between alluvium and bedrock, at the southern limit of planned Wall RW1 and the location of the existing Abutment 1, based on our subsurface investigation and archived Logs of Test Borings, was extrapolated and determined to be at an approximate elevation of 5.1 m. At the location of Boring RW1-B1, near the northern limit of planned Wall RW1, the alluvium and bedrock interface was found to be at an elevation of about 3.0 m. At about the contact between the alluvium and bedrock, an alluvial gravelly and cobbly zone of variable thickness was found. Thus, the SPT blow counts within that zone should be considered misleading. Bedrock consists of sandstone with claystone and siltstone interbeds. From the geotechnical engineering standpoint, based on core samples index tests and SPT blow counts, we found the bedrock to be competent. In addition, we found that claystone layers or interbeds could not be penetrated by the SPT sampler. For groundwater conditions at the location of Wall RW1, reference is directed to the section titled "Groundwater" on page 2 of this report.

FOUNDATION CONCLUSIONS AND RECOMMENDATIONS

The Proposed Retaining Wall RW1 was originally planned to be supported on a pile foundation using precast concrete driven piles Class 400 with a Design Loading of 400 kN, nominal Compressison of 800 kN and no Nominal Tension.

As indicated in the Subsurface Soil Conditions section of this report, the alignment of the proposed Retaining Wall RW1 is underlain by fill materials that in turn are underlain by alluvial soils. Some of these materials are of low relative density and are present below the groundwater table. During a seismic event these materials could potentially liquefy and cause damage to the wall. The damage, however, will be no greater than that for the existing freeway facilities including the approach ramp. In a view of this conclusion, and considering that the wall average height is relatively low, it is our recommendation that Retaining Wall RW1 be supported on spread footing bearing on a well-compacted pad built out of structural backfill material.

We recommend that along the proposed Wall RW1 alignment, 1.2 m thick layer of existing fill materials below the bottom elevation of the planned footing be removed and replaced with structural backfill compacted to 95% Relative Compaction in accordance with CTM Standard 216. The horizontal limits of the removal should extend a minimum 1.2 m beyond the edges of structural backfill compacted to 95% Relative Compaction in accordance with CTM Standard 216. The horizontal limits of the removal should extend a minimum 1.2 m beyond the edges of the proposed wall footing. In addition, Wall RW1 should be backfilled with structural backfill material in accordance with the standard specifications. This structural backfill should be benched into the existing slope in accordance with Caltrans Standard Specifications and compacted to 95% Relative Compaction in accordance with CTM 216. In addition, the top 0.3 m thick layer of the existing slope should be removed. The structural backfill material, when compacted to 95 % of Relative Compaction should yield strength parameters comparable to those shown for structural backfill in Table 2. Due to the fact that the temporary cut into the existing fill approach ramp should not be steeper than 1:1 (V:H), we anticipate that the recommended foundation improvement will involve temporary shoring along the southern section of the wall alignment (where the wall footing is the widest) during the construction of Wall RW1.

As previously stated, the foundation of the proposed Wall RW1 will be embedded in structural backfill material. Based on a footing embedment depth of 0.95 m, foundation width of 3.05 m, and the soil parameters presented in Table 2, we have calculated an allowable bearing capacity of 606 Kpa. The allowable soil bearing capacity is based on a safety factor of 3.0 for dead load plus live load. Per the 1999 Standard Plan B3-1 (Standard Retaining Wall Type 1), the design (applied) maximum toe pressure for a 5.5 m maximum high wall and Case I Loading (level plus 11.5 KPa surcharge) is 190 KPa. Therefore, the structural backfill pad along with its underlying fill materials will provide adequate bearing capacity for the proposed wall. Accordingly, the Standard Plan Design Wall Type 1 may be used for Wall RW1.

ABUTMENT 1

SITE CONSIDERATIONS

Abutment 1 of the Bridge No. 57-0884F lies at the northern limit of the Northeast Connector from southbound I-15 to eastbound I-8. Abutment 1 is located at approximate Station 121+31, where it bounds to the south the approach ramp of the aforementioned connector. Widening the approach ramp involves the widening of Abutment 1 to the east and construction of the proposed Retaining Wall RW1.

SUBSURFACE SOIL CONDITIONS

The locations of the proposed Retaining Wall RW1 and the existing Abutment 1 are adjacent to each other. Therefore, the description of the subsurface soil conditions at the location of Wall RW1 presented earlier in this report is applicable to the proposed Abutment 1 widening.

the approximate elevation of the alluvium and bedrock interface at the location of Abutment 1. In addition, it is our understanding that the preliminary plans were to support the addition to Abutment 1 on a pile foundation consisting of driven precast prestress concrete piles Class 625, with a Design Loading of 625 kN, Nominal Compression of 1250 kN, and no Nominal Tension.

Given the site conditions, it is our recommendation the addition to Abutment 1 be supported on 380 mm concrete Class 625 piles. We understand that your office has performed a lateral stability analysis for this pile type and found the results satisfactory. The recommended piles should be driven to the elevation of the interface between alluvium (gravelly and cobbly zone) and competent bedrock (sandstone with claystone interbeds) in order to attain the required design pile capacity. The total downdrag force for the pile in the event of liquefaction is estimated to be about 445 kN. These piles are primarily end-bearing. The pile data for the addition to Abutment 1 piles are presented in Table 3, below.

Table 3: Pile Data Table

LOCATION	PILE TYPE	DESIGN LOADING (kN)	NOMINAL RESISTANCE		DESIGN TIP ELEVATION (m)	SPECIFIED TIP ELEVATION (m)
			COMPRESSION (kN)	TENSION (kN)		
Abutment 1	Class 625	625	1250	0	4.9 [1,2]	4.9 [3]

Notes: Design tip elevation is controlled by the following demands: 1) Compression; 2) Liquefaction Potential exists from elevation 15.08 m to approximate elevation 4.9 m; 3) Specified Tip Elevation shall not be raised.

CONSTRUCTION CONSIDERATIONS

- Hard driving through the gravelly and cobbly zone should be anticipated. Due to the cobbly and gravelly nature of the interface between the alluvium and bedrock, the proposed piles may reach refusal before specified tip elevation. Piles that reach refusal within 1.2 m of specified tip elevation are considered good and may be accepted subject to approval by the Resident Engineer. A pile has reached refusal when 2x design loading is attained.

- Assessment of pile capacity is based on the ENR equation indicated in Section 49-1.08 of the Standard Specifications.

- Piles that reach refusal within 1.2 m of the specified tip elevation and are adequately seated may be cut off to the designed top pile elevation. The acceptable cut off elevation is subject to the written approval of the Resident Engineer.

- Piles to be driven through embankment fill could be driven in oversized drilled holes in conformance with the provisions in Section 49-1.06, "Predrilled Holes," of the Standard Specifications at the location of Abutment, subject to the written approval of the Resident Engineer.

- Jetting or drilling to obtain the specified penetration in conformance with the provisions in Section 49-1.05, "Driving Equipment," of the Standard Specifications shall not be used for driven type piles.

If you have any questions or comments regarding this report, please call Jeff Tesar at (858) 467-2716 (Calnet 734-2716) or Zia Yazdani at (858) 467- 4054 (Calnet 734-4054).

Prepared by Jeff Tesar, March 18, 2002

J. Tesar

Jeff Tesar
Associate Engineering Geologist

Zia Yazdani

Zia Yazdani
Associate Materials and Research Engineer

Geotechnical Branch 11

REGISTERED GEOLOGIST
JEFF J. TESAR
No. 2137
CERTIFIED
ENGINEERING
GEOLOGIST
STATE OF CALIFORNIA

REGISTERED
No. 2119
GEOTECHNICAL
STATE OF CALIFORNIA

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JT

cc:

James Chai
Joe Egan
Homayoon Gerami
RGES.02